ROARK

Investigation of an Electric Railway Viaduct

Civil Engineering

B. S.

1911

INVESTIGATION OF AN ELECTRIC RAILWAY VIADUCT

BY

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THESIS

FOR THE

DEGREE OF

BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

IN THE

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1911

UNIVERSITY OF ILLINOIS

May 25, 1911

I recommend that the thesis prepared under my supervision by RAYMOND JEFFERSON ROARK entitled Investigation of an Electric Railway Viaduct be approved as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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Contents

An Investigation of an Electric Railway Viaduct

Character of the Investigation.

The investigation of this viaduct is made with a view of determining whether or not the structure is capable of carrying, with safety, the loads to which it is subjected. The present physical condition and serviceability of the structure are also considered.

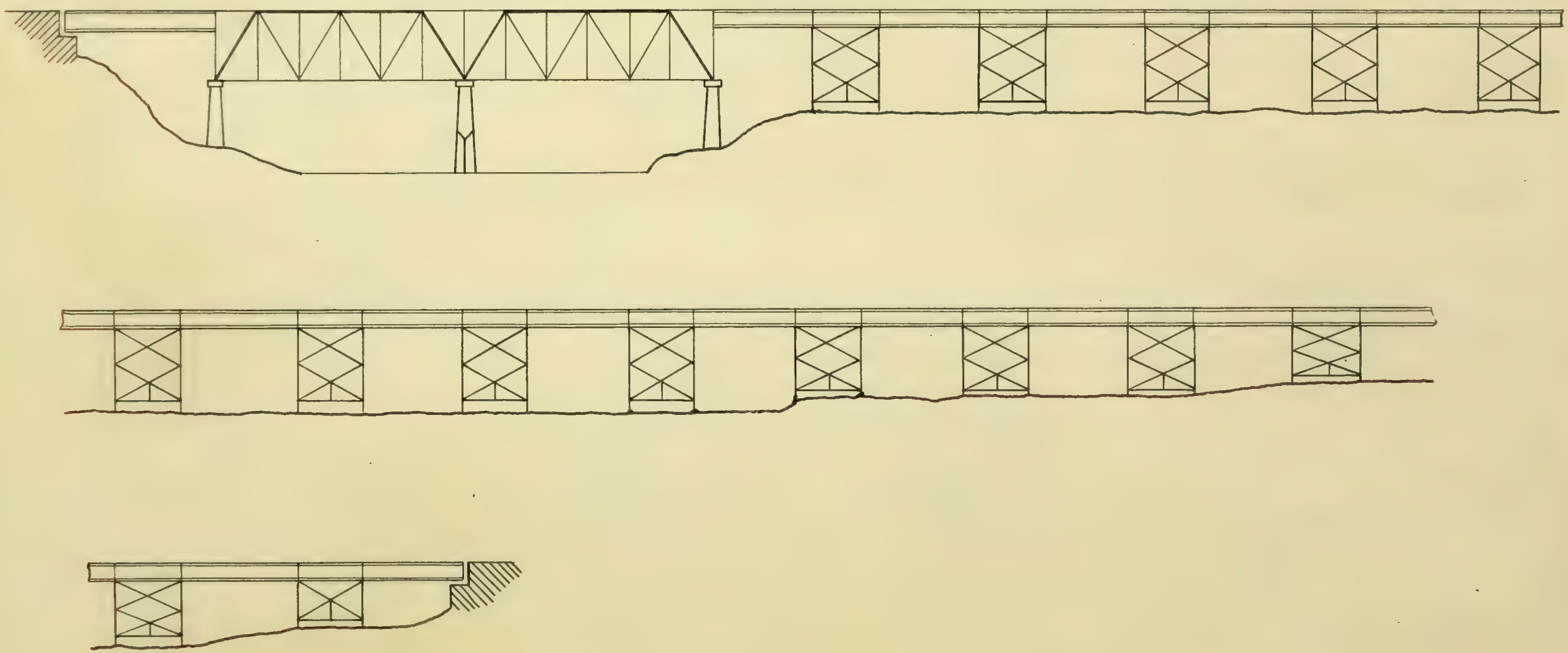
General Description.

The viaduct is on the Catlin division of the Illinois Traction System, and crosses the valley of the Vermilion river near the southern edge of Danville, Illinois.

A general outline of the entire structure is shown on the accompanying diagram, Fig. I., Plate I. From left to right it consists of one deck girder of seventy-five foot span, two deck trusses of the Warren type, each of of one hundred and twenty-six foot span, and thirty-two deck girders consisting alternately of fifty foot and of thirty foot spans, giving a total length of sixteen hundred and seven feet between centers of end bearings. The viaduct is on a tangent, but the track enters on a curve at both ends. The right half of the trestle is on a slight grade, sloping upward from left to right. The maximum height

2 Fig I

THE VIADUCT



Girder 1 Abutment to truss.
Girder 2 Between towers.
Girder 3 Across towers.

PLATE I

of the track is about seventy feet above mean water level.

The viaduct was built by the Indiana Bridge Company in 1906.

Method of Investigation.

The method of investigation was as follows.

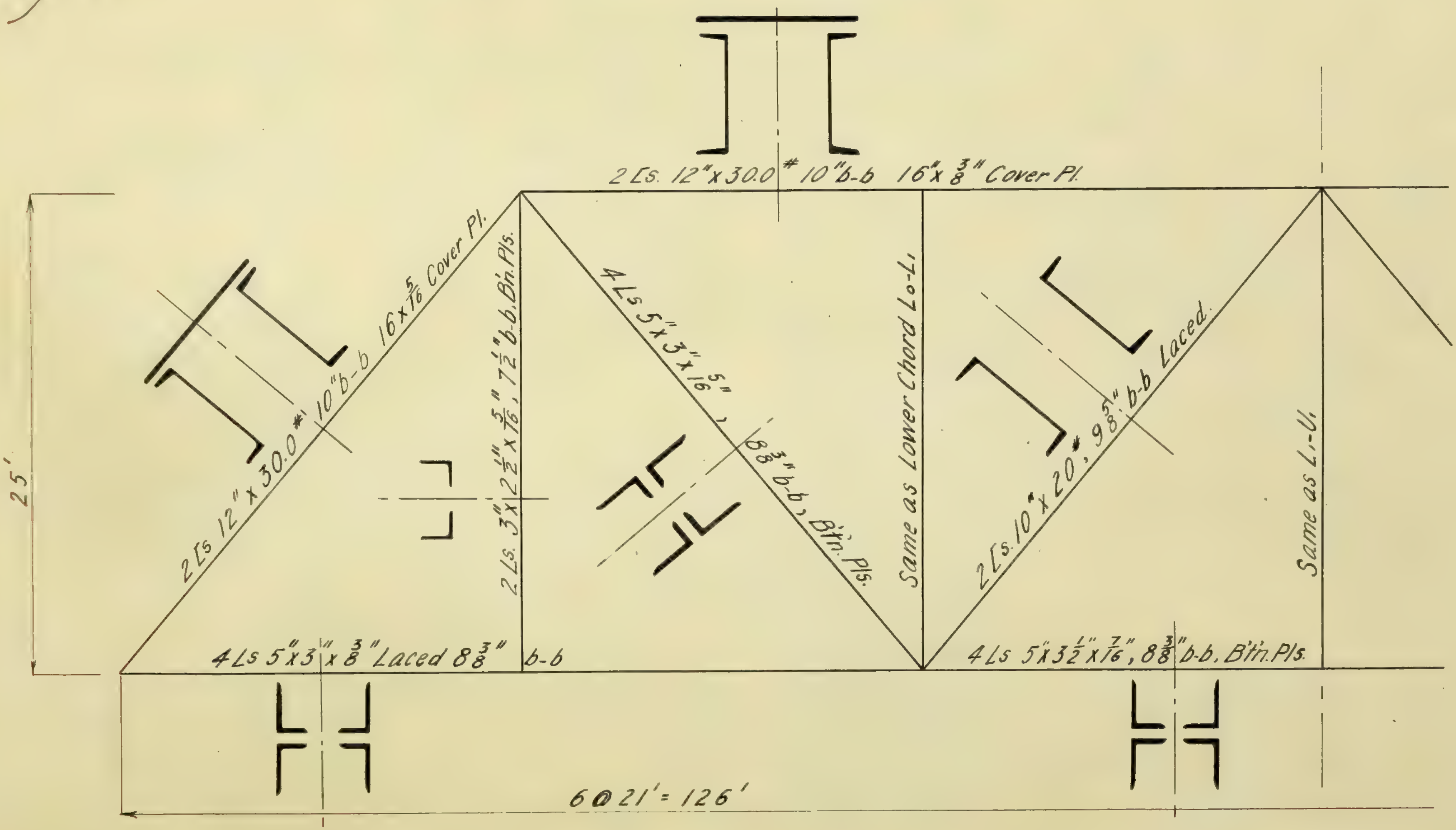
All of the various members and parts of the girders and trusses were carefully measured and their dimensions recorded on sketches showing their arrangement and make-up. The depth of the girders, the dimensions and the riveting of the flange angles, stiffeners, lateral and cross bracing, and the spans center to center of bearings was determined. All joints and connections were investigated and the bearing areas of pedestals and bed-plates determined. At the same time, the physical condition of the materials and parts was carefully noted. Fig II shows the make-up of all truss members, and Figures IX, X and XI show the composition of the girders.

Sufficient data having been secured, the weights of the various parts were calculated and the dead loads deduced therefrom. The entire weight of each structure, i.e; each girder and truss, was reduced to a uniform load and considered as all acting on the upper chord. The weights obtained are tabulated below.

Girder No. 1. Total weight = 36700. lb.
Dead load = 682. lb./ft. span.

Fig. II

MAKE-UP OF TRUSS MEMBERS



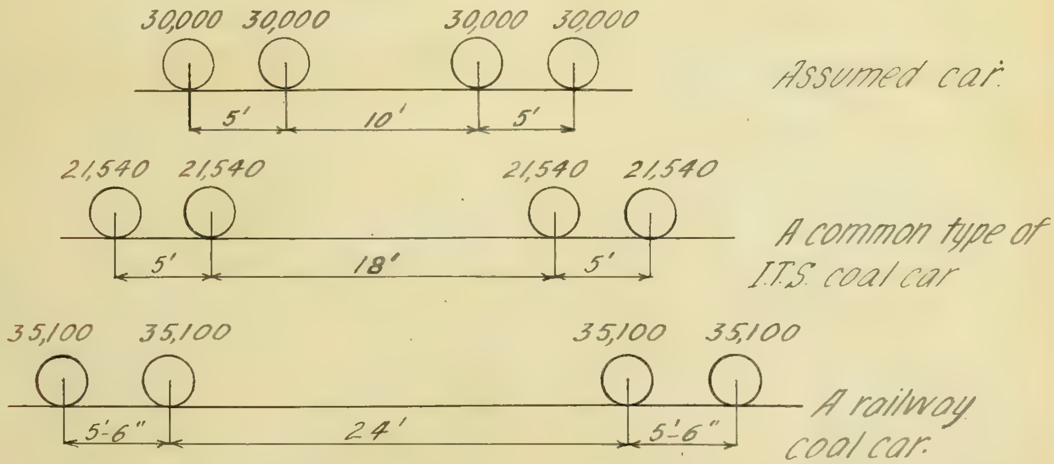
Total weight = 84,540.#
Weight per lin. ft. = 1000# including track

PLATE II

Truss	Total weight	= 84540 lb.
	Dead load	= 1000 lb/ft. span
	Panel load	= 10500 lb.
Girder 2.	Total weight	= 17100 lb.
	Dead load	= 538 lb/ft. span
Girder 3.	Total weight	= 4440 lb.
	Dead load	= 344 lb/ft. span

From these loadings the dead load stresses were calculated.

In computing the live load stresses in the structure a wheel loading was assumed and this reduced to an equivalent uniform load. The loading assumed is shown below, together with that caused by the heaviest cars which are ordinarily used on the roads of the Illinois Traction System. In the assumed loading the cars



weigh one hundred and twenty thousand pounds each, the axles being five feet center to center and the trucks fifteen feet center to center as shown. When reduced to an equivalent uniform load according to the proportions given in

Ketchum's specifications, this gave three thousand nine hundred and forty pounds per foot of span, or nearly the same as the uniform load for an E 40 loading. For the girders the uniform loading obtained was three thousand nine hundred pounds per foot for Girder 1, and four thousand two hundred pounds per foot for Girders 2 and 3. It is evident from the diagrams on the preceding page that the assumed loading is heavier than that ordinarily coming upon the viaduct.

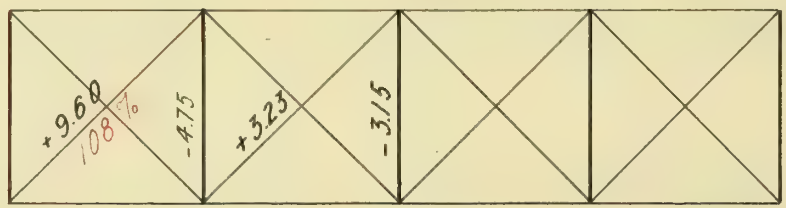
The loading having been determined, the maximum live load stresses were calculated and combined with the dead load stresses to give maximum stresses. These are shown on Figure III, Plate II for the truss. Impact was not calculated, this being provided for by the unit stresses used in computing the efficiencies.

Wind load stresses were calculated on the assumption of one hundred and fifty pounds per foot on the lower chord, and three hundred pounds per foot on the upper chord, half of this latter being considered as moving load. The wind stresses in the lateral bracing of the truss are shown on Plate III, page 7, and those in the bracing of the girders on Plates VII, VIII and IX, pages 17, 19, and 21. The stresses in the main members of the truss due to wind were found to be negligible.

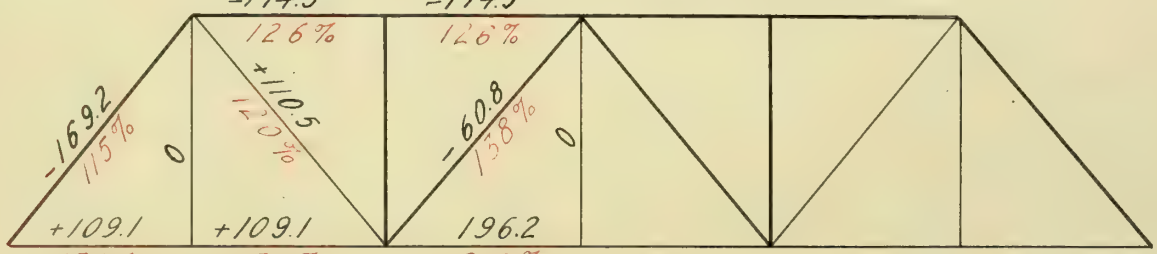
The stresses due to combined wind and train loads on the trestle towers were computed

Fig. III

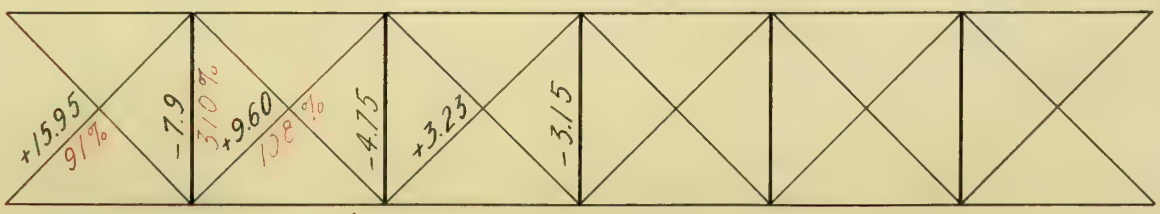
MAX. STRESSES AND EFFICIENCIES



TOP LATERAL



TRUSS



LOWER LATERAL

PLATE III

and are given on page 24.

Efficiencies.

The stresses having been calculated, the efficiencies of all parts, members, and connections of the viaduct were computed, Ketchum's specifications being used. The method of calculating efficiencies is illustrated by the following example.

Member L_0-L_1, net area 10.32 □"

Dead Load stress = +22050, Req. area 0.89 □"

Live Load stress = +87000, Req. area 6.95 □"

Total Req. area 7.84 □"

Efficiency = Actual area ÷ Req. area

$$= \frac{10.32}{7.84} = 131\%$$

For the truss the efficiencies of all members are shown, together with the maximum stresses, on Plate III. The efficiencies of all the connections of main members are given on page 11, and the pedestal at L_0, and the connection of members at that point, are shown on Plate IV, page 10. The efficiencies of the floor members are given on page 13, and Plate V page 12, shows their arrangement and connection The investigation of the frame supporting the right hand end of Girder 1 is given on page 16. Plate VI, page 15, shows the makeup of this frame. The efficiencies of the various parts of Girders 1, 2, and 3 are given on pages 18, 20, and 22 respectively

and the makeup of the girders are shown in Plates VII, VIII, and IX, pages 17, 19 and 21. On page 24 are given the efficiencies of the members and joints of the trestle tower of greatest height, while Plate X, page 23, illustrates the same.

The unit stresses used in determining efficiencies are given below. They are taken from Ketchum's specifications, articles 36 a to 42 a.

Tensile stresses.

Wind bracing	18,000 lb/sq. in
Floor beams and stringers	13,000
Girder flanges	13,000
Main truss members	12,500 L.L., 25,000 D.L.

Compressive stresses.

Chord segments

$$P = 12{,}000 - 55\frac{l}{r}\ (L.L.),\quad P = 24{,}000 - 110\frac{l}{r}\ (D.L.)$$

Posts of deck bridges.

$$P = 11{,}000 - 40\frac{l}{r}\ (L.L.),\quad P = 22{,}000 - 80\frac{l}{r}\ (D.L.)$$

Rigid bracing.

$$P = 13000 - 60\frac{l}{r}\ (Wind).$$

Bearing on rivets.

Shop rivets 18,000 lb/sq. in. in main members.
80% of this in floor system.
140% of this in lateral bracing.

Field rivets
Two thirds of the above values.

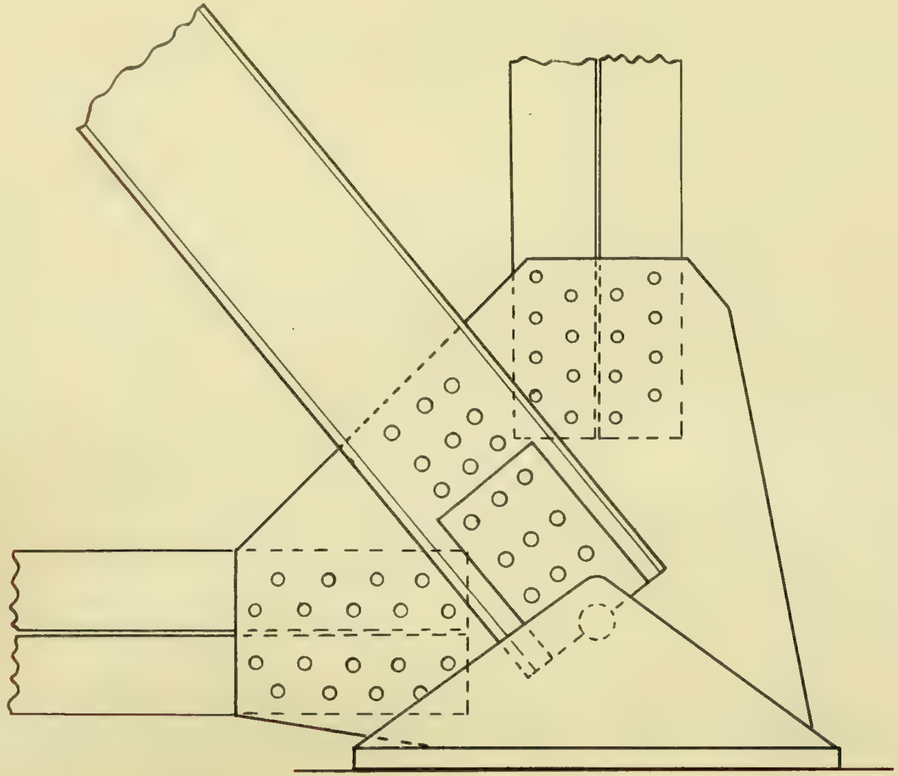
Shear on rivets.

Shop rivets 10,000 lb/sq. in. in main members.
Modified as above for other members.

Field rivets two thirds of above values.

Fig. IV

CONNECTION AT Lo



Gusset plate $\frac{1}{2}$" thick
Pin plate $\frac{3}{8}$" thick.
Pin $4\frac{1}{2}$" diam.

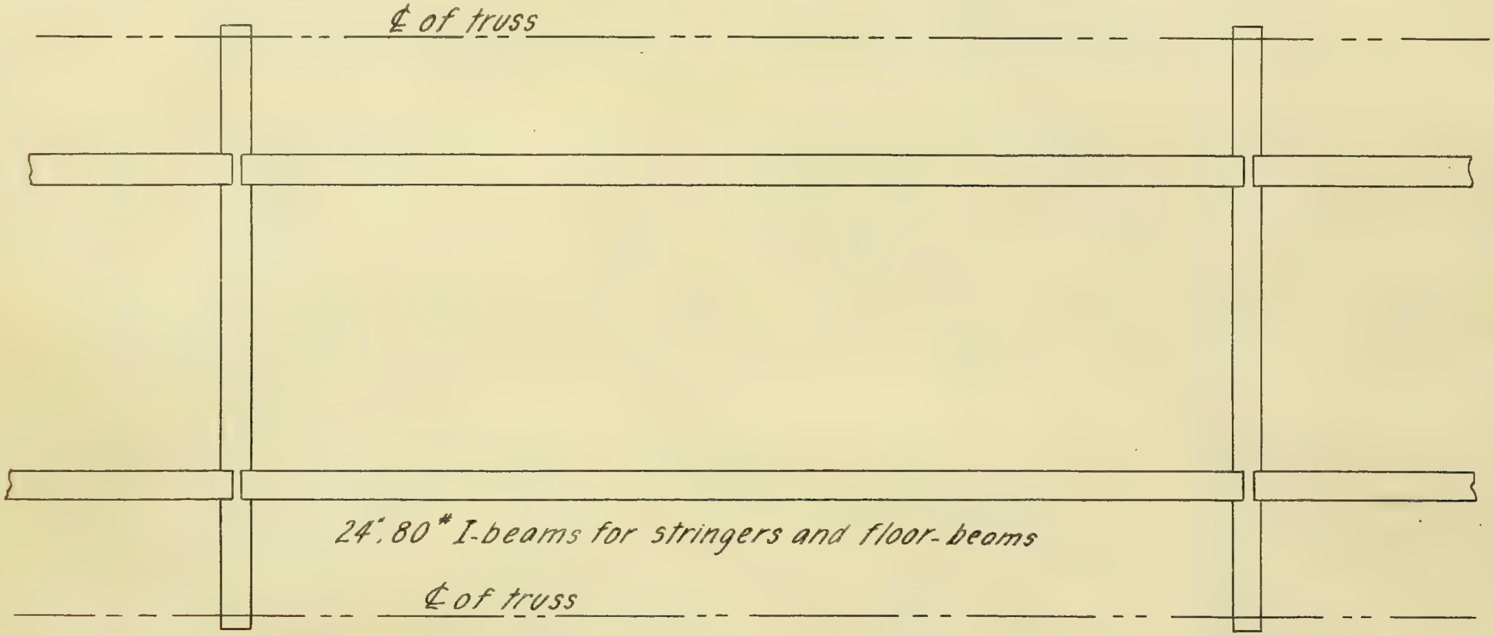
PLATE IV

CONNECTIONS of TRUSS MEMBERS

Connection at:	Member	Stress in Lb.	No. of Rivets	Efficiency.
L_0	L_0-U_1	169200	36 s.	146%
	L_0-L_2	109100	36 f.	135%
	Vertical	108000	32 f.	118%
U_1.	L_0-U_1	169200	40 s.	143%
	L_2-U_1	110500	32 f.	117%
	U_1-U_2	174300	58 f.	133%
L_2.	L_1-L_2	109100	36 f.	128%
	U_1-L_2	110500	32 f.	117%
	U_2-L_2	51900	20 f.	155%
	U_3-L_2	60800	16 f.	106%
	L_3-L_2	196200	52 s.	158%
U_2.	U_2-L_2	51900	16 f.	124%
U_3.	U_2-U_3	174300	30 f.	68%
	L_2-U_3	60800	16 f.	106%

 Fig. V

FLOOR SYSTEM OF TRUSS



Connection.

PLATE V.

FLOOR SYSTEM

Stringer:

Max. Moment	= 1,515,600 inch/lb.
Efficiency	= 149 %

Floor-beam:

Max. Moment	= 1,387,300 inch/lb.
Efficiency	= 158 %

Connection:

Reaction	= 21,700 lb.
No. rivets	= 11
Efficiency	= 213 %

Ties :

Efficiency	= 82 %

ROLLER BEARINGS OF TRUSS

Rollers:

Total reaction = 257150 lb.

Efficiency in bearing = 250 %

Expansion:

Allowance made = 3 inches.

Allowance required = $1\frac{3}{4}$ inches.

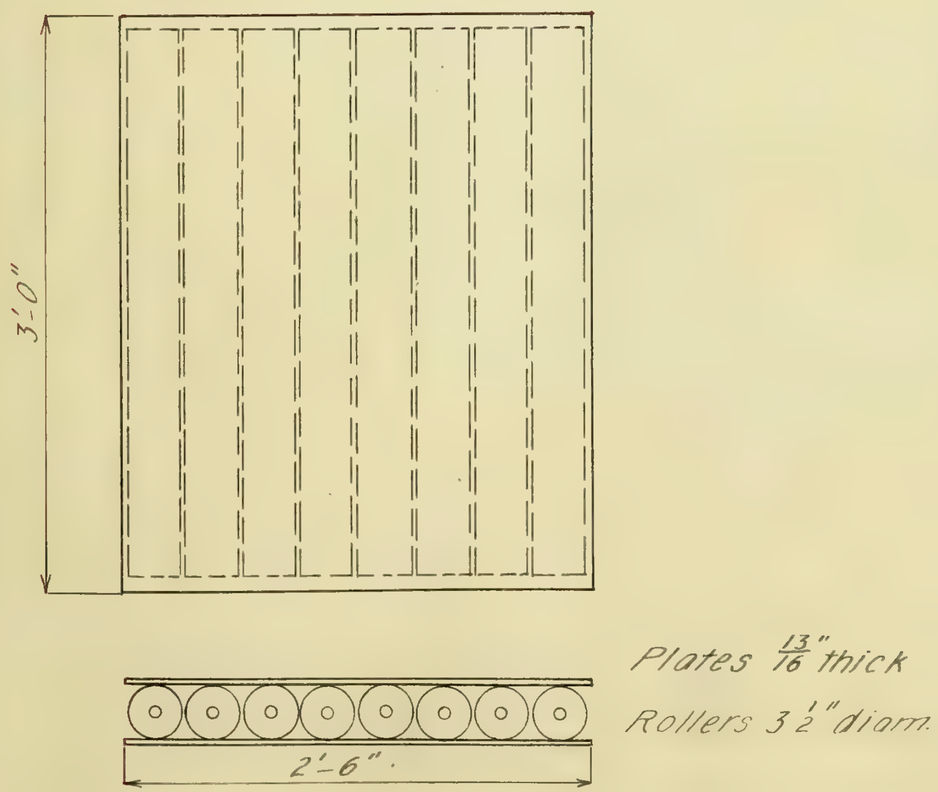


Fig. VIII.

Fig. VII

GIRDER SUPPORT

RIGHT-HAND END G1

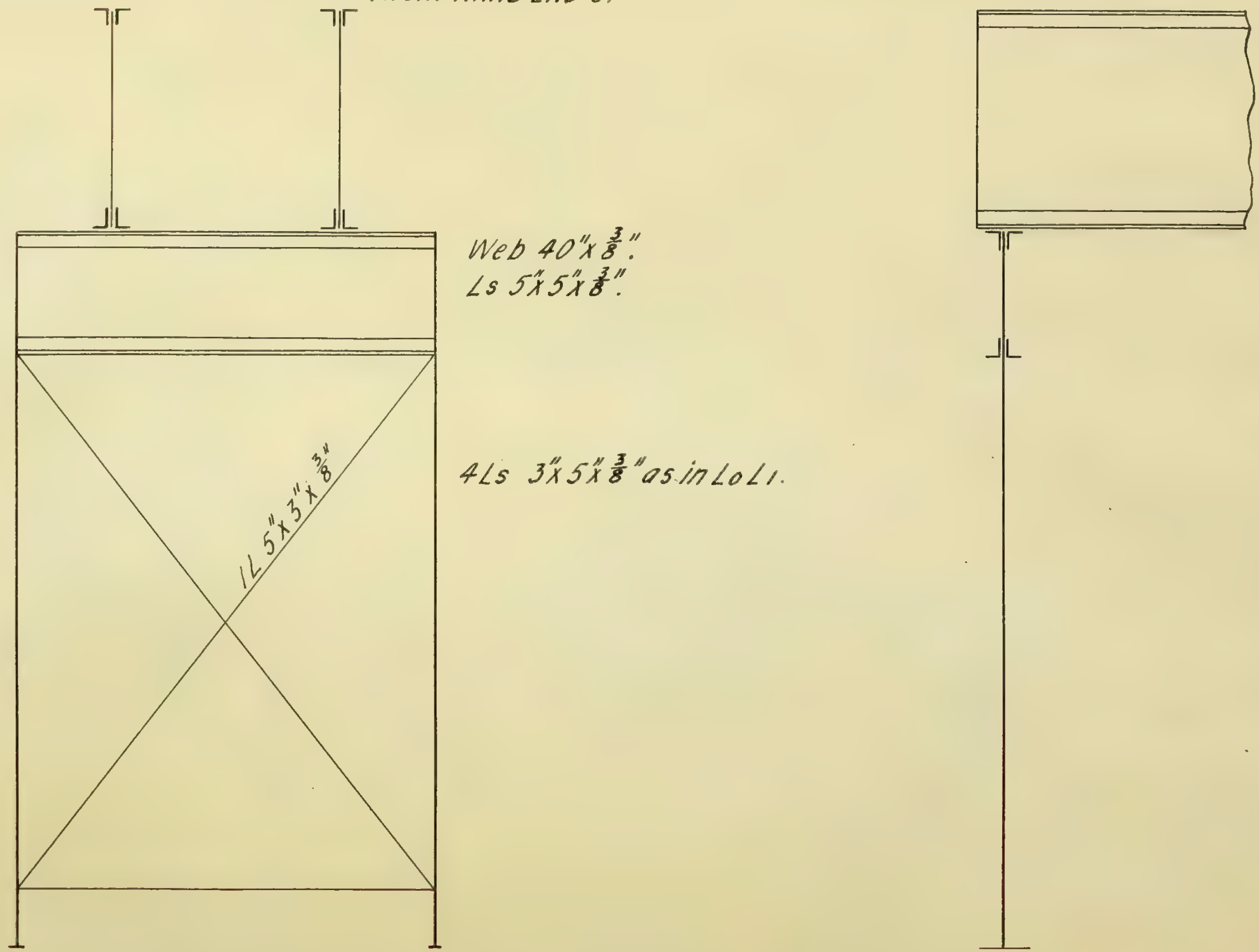


PLATE VI

GIRDER SUPPORT
(Right hand end G_1)

Verticals:

Live load stress	=	93,900 lb.
Dead load stress	=	18,000 lb.
Efficiency of member	=	84.5 %

Cross Girder:

Max. moment	=	3,690,000 in. lb.
Efficiency of girder	=	90.3 %

Connection girder to post:

Stress on 30 rivets	=	111,900 lb.
Efficiency of rivets	=	105 %

Fig. IX

GIRDER No. 1.

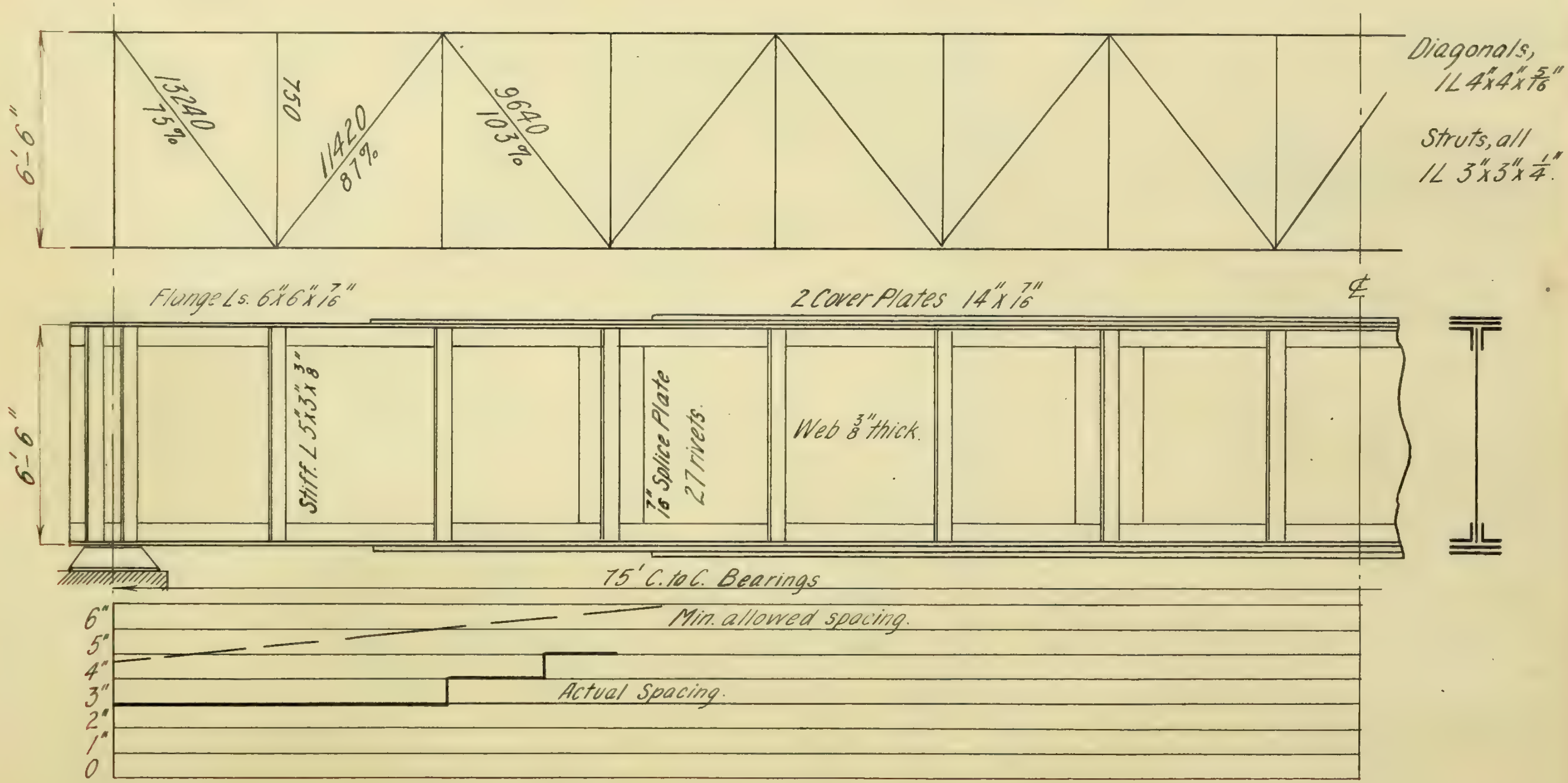


PLATE VII

GIRDER I

Span 75'

Live load 3900 lb. per ft of span

Dead load 682 lb. per ft. of span

Flange:

Max. moment	=	16,150,000 inch/lb.
Efficiency of flange	=	119%

Web:

Max. shear	=	86000 lb.
Unit stress on web	=	2940 lb. per sq. inch.

Splice Plate:

Total shear	=	545 lb. per s
Unit stress on Plate	=	1900 lb. per sq. inch.

Spacing of Flange rivets:

As shown on Plate VII

Wind Bracing:

As shown on Plate VII

Pedestal:

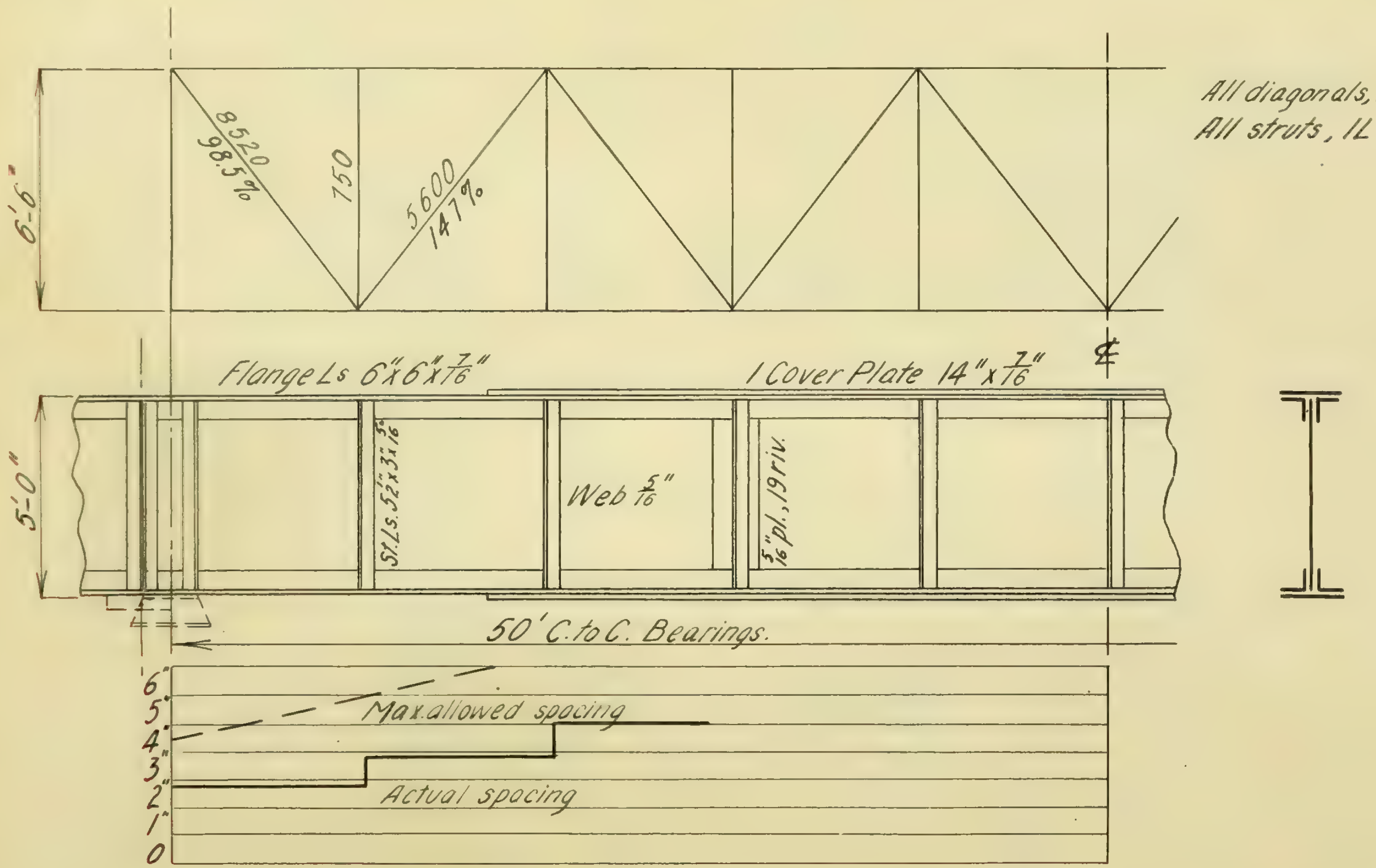
Unit bearing stress on concrete 200 lb/sq. in.

Ties:

As in truss.

Fig. X

GIRDER No. 2.



All diagonals, 1L 4"x4"x5/16".
All struts, 1L 3"x3"x¼".

PLATE VIII.

GIRDER 2.

Span 50'

Live load 4200 lb. per ft. of span.

Dead load 538 lb. per ft. of span.

Flange:

Mor. moment = 14,800,000 inch lb.

Efficiency of flange = 143 %

Web:

Max. shear = 59000 lb.

Unit stress on web = 3180 lb per sq. in.

Splice Plate:

Total shear =

Unit shear on plate = 2060 lb per sq. in.

Efficiency of rivets = 350 %

Spacing of Flange rivets:

As shown on Plate VIII

Wind Bracing:

As shown on Plate VIII

Pedestal:

Unit bearing on concrete 137 lb/sq. in.

Fig XI

GIRDER No. 3.

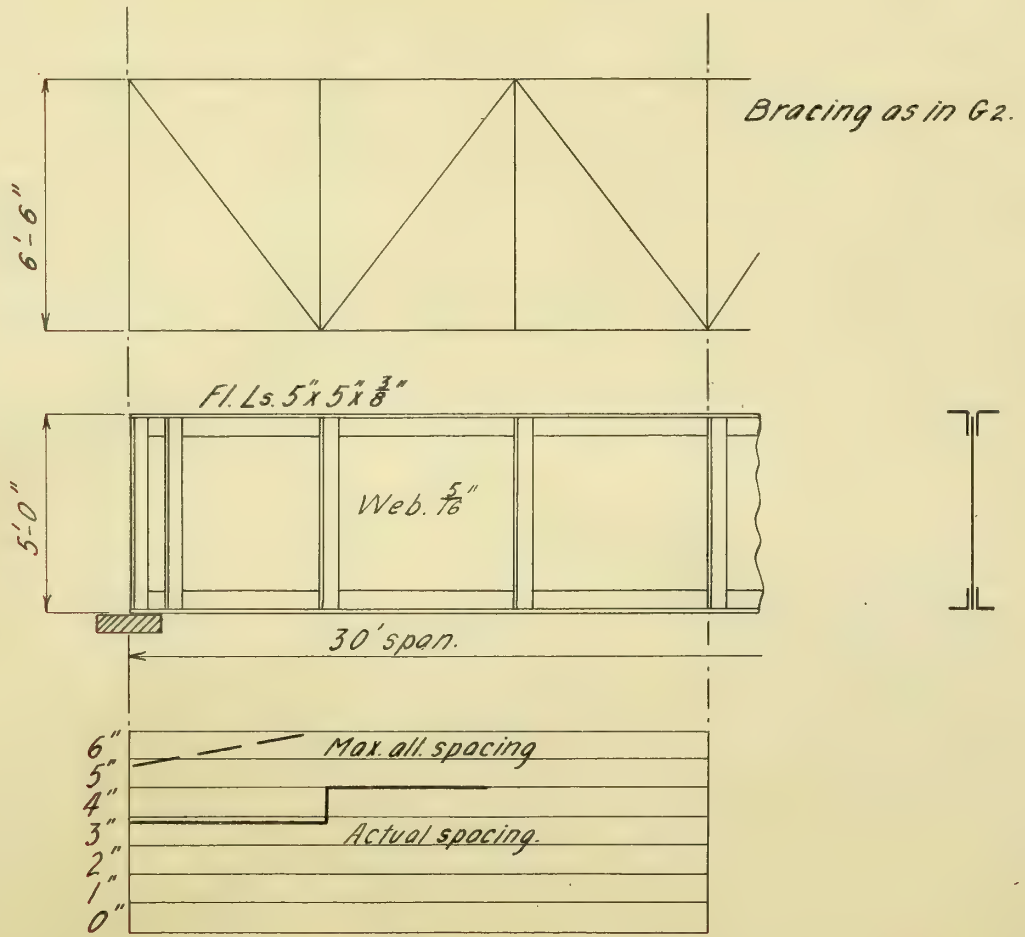


PLATE IX

GIRDER 3.

Span 30'

Live load 4200 lb. per ft. of span

Dead load 344 lb. per ft. of span.

Flange:

Max. moment = 3,080,000 inch lb.

Efficiency of Flange = 153 %

Web:

Max. shear = 35400 lb.

Unit stress on web = 2370 lb. per sq. in.

Splice Plate:

As for Girder 2. Safe.

Spacing of Flange rivets:

As shown on Plate IX

Wind. Bracing:

As shown on Plate IX

23 Fig. XII

TRESTLE TOWER

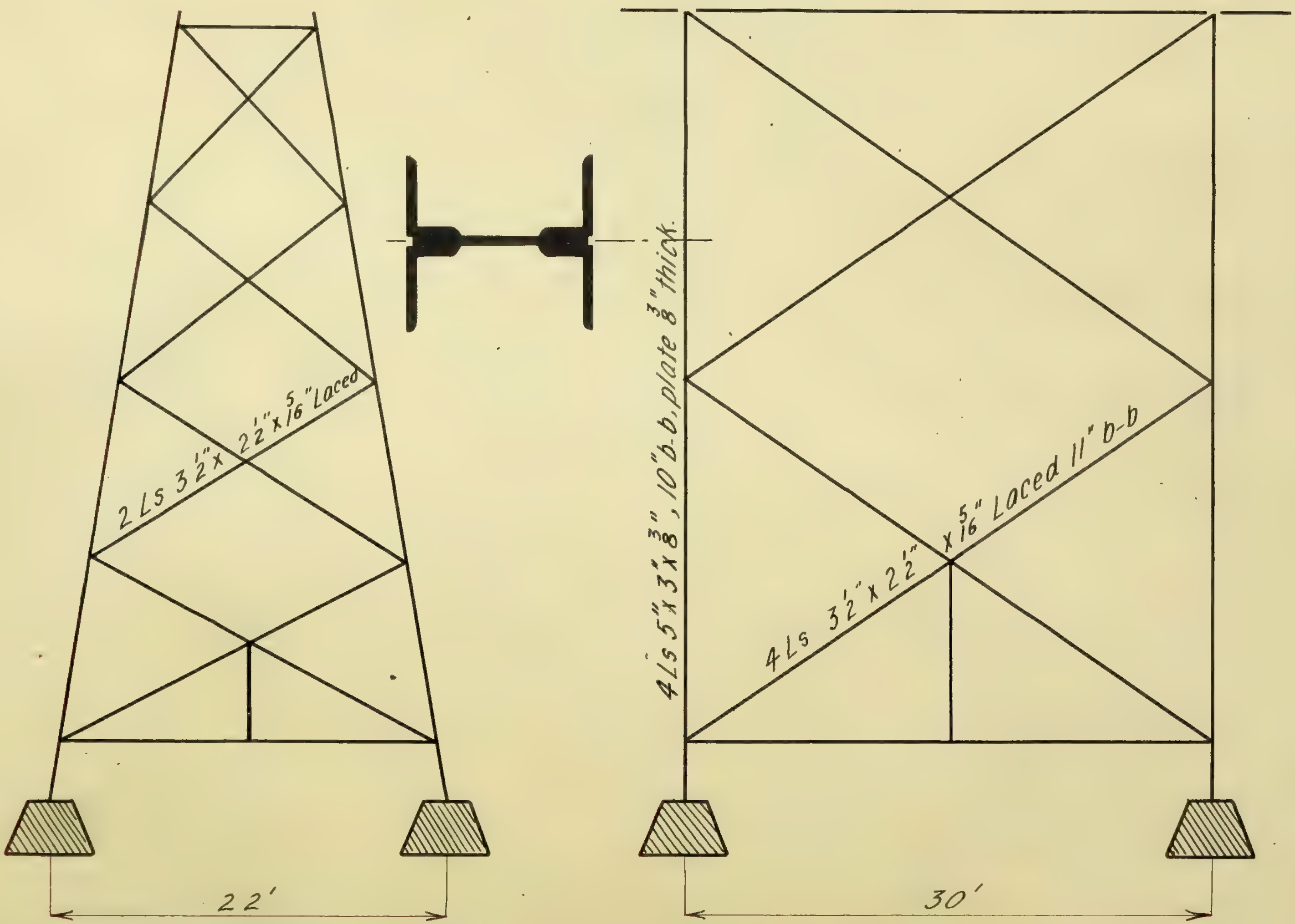


PLATE X.

TRESTLE TOWER

Main Columns:

Max. combined stress = 141000 lb.

Efficiency of member = 96 %

Transverse Bracing:

Efficiency of members = 75 %

Efficiency of connections = 22 %

Long. Bracing:

Efficiency of members = 102 %

Efficiency of connections = 25 %

Pedestals:

Bearing on concrete = 325 lb. per sq. in.

Efficiency of anchorage = 178 %

Fig. XIII shows connection of transverse bracing.
Fig. XIV shows connection of longitudinal bracing.

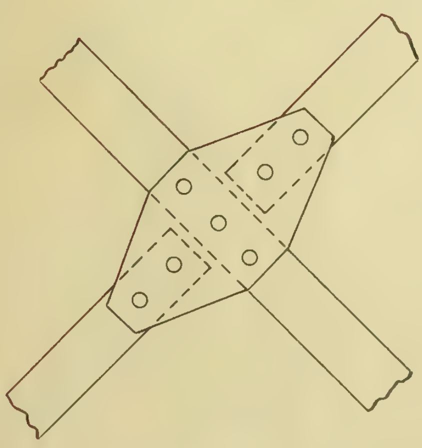


Fig. XIII

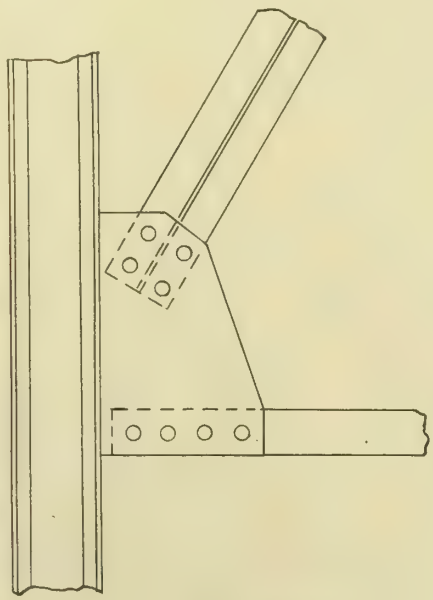


Fig. XIV.

Discussion.

It is evident, from the efficiencies of the various members and connections, that the viaduct is strong enough to sustain, with perfect safety, the heaviest loads which are likely to come upon it. Two or three members appear to be inadequate, but it should be borne in mind that the loading assumed in this investigation is heavier than that ordinarily carried.

Some members appear to be considerably more efficient than others, but these differences are probably due in part to differences in the specifications used for design and for investigation. On the whole a high efficiency obtains thruout the structure. The efficiencies of the girders are: for Girder 1, Girder 2, and Girder 3; 119%, 143%, and 153%, respectively. For the truss, the efficiencies of various members vary from 138% to 91.4%. The weakest member is the center length of the lower chord, L_2-L_4. All others are over 100%. Perhaps the most serious weakness in the entire structure is the low efficiency of the verticals which support the right hand end of Girder 1, the investigation of which member is given on page 16. They take the full end reaction of the girder, and in addition a half panel load of the truss, under which loading they have an efficiency of but 84.5%. These members should undoubtedly be heavier.

The connections are of lower efficiency than

the main members in the truss, and in the wind and tower bracing are quite inadequate to develop the full strength of the members. In the truss, however, all of the connections are strong enough to be safe. The efficiencies of the connections are calculated from the stresses to which they are subjected, and not from the full strength of the members.

The stringers and floor beams of the truss are ample, as are also their connections. The ties, however, are inadequate thruout. This is not because of their size, but on account of their unusually large spacing, — eighteen inches center to center, or thirteen inches clear.

As has been said, certain connections in the subordinate members of the structure are altogether inadequate. Thus, in the portal bracing of the truss only four rivets are used to connect members consisting of two angles four by three by five sixteenths inches. The efficiency of this connection, based on the full strength of the members, is only 30%. Again, in the bracing of the trestle towers the efficiency of the connections of the transverse bracing is 22% and of the connections of the longitudinal bracing 25%. These connections are illustrated by Fig. XIII and Fig. XIV, page 24. Connection is made to but one leg of the angles in the wind bracing of both truss and girder spans, but the members are for the most

part strong enough, even when the section of the riveted leg only is considered. It is very improbable that the members composing the wind and tower bracing will ever be stressed as high as is herein calculated, but at the same time such connections as those noted above are utterly inexcusable.

A few other minor points of design are open to criticism. For instance the top cover plates on Girders 1 and 2 do not extend the full length of the span, and no cover plate at all is used on Girder 3. As a consequence some rusting and corrosion of the upper edge of the web is apparent at points thus left unprotected. The rivet spacing in the flanges of the girders is irregular, the spacing not increasing uniformly from the ends toward the center. Another weak feature of the girders is the absence of any lower wind bracing.

In spite of the faults mentioned above, the viaduct is, on the whole, very good both in design and in construction. The behavior of the structure under traffic is indicative of great strength and rigidity, there being practically no noticeable vibration or deflection during the passing of heavy cars.

The viaduct is in good physical condition, the only parts which seem to have suffered material deterioration being several of the lower cross-braces of the trestle towers, which

are considerably rusted. The lower flanges of some of the girders are also rusted in places, but not to any serious extent. The masonry of the abutments, piers, and pedestals is in excellent condition.